

Challenges In Design And Construction Of Deep Excavation

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ABSTRACT: Due to scarcity of land, especially in urban areas, the need for basements to optimise the use of land has resulted in construction of deep excavation works. Deep excavation works pose great challenges to geotechnical engineers, particularly in complex ground conditions such as in limestone formation, where it exhibits notorious karstic features with irregular bedrock profiles, variable weathering condition, cavities and slime zones. With proper geotechnical input, costly failures and delays associated with underground works such as excessive groundwater lowering, occurrences of sinkholes, excessive ground settlements, etc. can be prevented. Suitable temporary earth retaining system and rock strengthening works have to be properly designed to prevent such failures. This paper presents design principles of temporary earth retaining stabilising system (ERSS) together with vertical rock excavation as well as three case studies including from Mass Rapid Transport (MRT) projects, with various challenging geological formations found within the Klang Valley of Malaysia; Limestone and Alluvium with high ground water table.

Keywords: Deep excavation; temporary earth retaining stabilising system, ERSS, limestone; Kenny hill, alluvium

1. INTRODUCTION

With the expansion of underground transportation systems such as the Klang Valley Mass Rapid Transit (KVMRT) in Malaysia, various underground stations, tunnel escape shafts, tunnel intervention and ventilation shafts need to be constructed. These constructions involve deep excavation works. Deep excavation works pose great challenges to geotechnical engineers, particularly in complex ground conditions as well as in close proximity of existing buildings in urban areas. Careful selection of suitable retaining walls and support systems is crucial, taking into consideration such as control of ground movement, lowering of the groundwater table, encroachment into neighbouring land, etc. The design of temporary earth retaining stabilising system (ERSS) requires meticulous analyses and evaluations of various possible failure modes, such as overall stability, basal heave failure, hydraulic failure, structural failure, etc. This paper presents design principles and lessons learnt from temporary earth retaining stabilising system (ERSS) together with vertical rock excavation as well as three case studies including from Mass Rapid Transport (MRT) projects, with various challenging geological formations found within the Klang Valley of Malaysia; Limestone and Alluvium with high ground water table.

2. DESIGN CONSIDERATIONS

Major design considerations for deep excavations revolve within the following aspects:

- a) Overall stability
- b) Basal heave failure
- c) Hydraulic failure
- d) Axial stability
- e) Finite element analysis
- f) Impact on buildings from ground movement associated with excavation

It is vital that preliminary analyses be carried out for various options of walls and support systems to assess on the cost and time of construction together with technical requirements on safety and its influence on adjacent structures before the selection of the final option to produce safe and economical design.

This paper updates some of the recent developments for the design and construction of deep excavation works in Malaysia. The flowchart for analysis and design of deep excavation works is updated from Tan et al. (2016), as shown in Figure 1. The figure shows the routine from the beginning such as subsurface

information needed for analysis, design and finally producing construction drawings.

2.1 Planning of Subsurface Investigation and Soil Parameters

Proper planning and supervision of subsurface investigation (SI) are of utmost importance to the designer in order to produce a safe and economical design for a deep excavation. A thorough discussion of the planning of SI, field and laboratory tests may refer to publications by Geotechnical Control Office of Hong Kong (GEOGUIDE 2: Guide to Site Investigation, 2017) and Clayton et al. (1995). Generally, the following soil parameters should be obtained from the SI:

- a) Shear strength parameters of soil (ϕ' and c')
- b) Stiffness of soil (E')
- c) Permeability of soil (k)
- d) Groundwater level

The above information is usually obtained from routine SI programmes except for soil stiffness which requires special testing techniques and interpretation of results. The use of pressure meter tests is recommended to obtain representative soil stiffness values for design. Further discussion of the use of appropriate soil stiffness values will be presented in the next section.

2.1.1 Shear Strength Parameters

The effective shear strength parameters of soil (ϕ' and c') are commonly obtained from Isotropically Consolidated Undrained Triaxial (CIU) Test with pore pressure measurements. If finite element method is used, understanding of the constitutive models and numerical algorithms adopted in the finite element software is important in order to model the problem appropriately. The following are recommended:

- a) Hardening soil model should be used to model excavation problems, as the conventional Mohr-Coulomb model is unable to model unload-reload problems properly. Mohr-Coulomb model is based on elastic behaviour and is unable to model density and shear hardening which renders it inaccurate for deformation problems.
- b) Modelling of excavation is recommended to be performed in effective stresses and with effective stiffness and strength parameters, if possible.

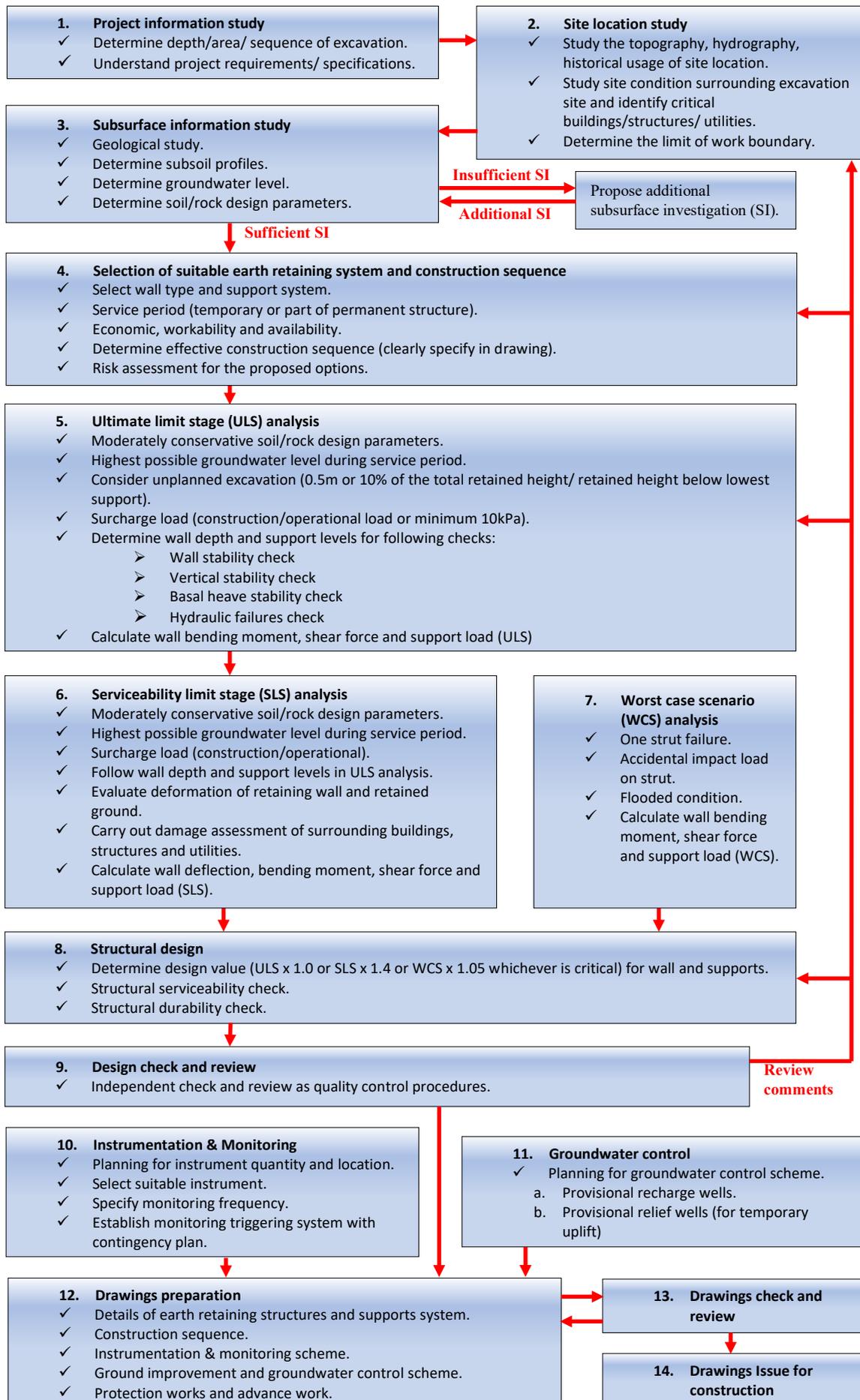


Figure 1 Flowchart for analysis and design of deep excavation works modified after Tan et al. (2016)

information on effective strength parameters is not available, undrained strength parameters ($c = c_u, \phi = 0, \psi = 0$) with effective stiffness parameters can be used. Proper understanding of the constitutive soil models is essential.

2.1.2 Soil Permeability

Soil permeability (k) is important to ensure the drained or undrained behaviour of soil is modelled correctly. In-situ tests are recommended in order to account for the complex soil stratigraphy at site which may not be accurately measured in a laboratory. Either rising, falling or constant head tests can be carried out in-situ. The values obtained should be compared to published values as a check to ensure the values obtained are reasonable for a given soil condition. Figure 2, reproduced from BS8004: 1986, is useful as a simple check.

2.1.3 Soil Stiffness & Small Strain Stiffness

Empirical correlations are commonly used for estimation of soil stiffness. This is because routine laboratory tests give soil stiffness parameters that are significantly less than the stiffness values derived from back analysis of field measurements. This is primarily due to disturbance to the soil samples and also testing at strain levels which are larger than the range which is appropriate for retaining walls. This is illustrated in Figure 4 (Atkinson, 2000) which shows the strain dependent characteristics of soil stiffness, is non-linear. It can also be seen that typical ranges of strain for conventional laboratory testing are larger than the range for retaining walls.

Though various empirical correlations are available to determine small-strain stiffness for design, it should be noted that empirical correlations are highly dependent on factors such as local soil conditions.

For practical purposes, small-strain stiffness is probably most reliably obtained using geophysical techniques through field and laboratory, in which shear wave velocity can be measured as shown in Figure 5, after Sabatini et al. (2002). Cross hole seismic test is preferred as it measures the average shear wave velocity laterally along the soil layers between two boreholes while the downhole seismic test yields a shear wave velocity vertically across different soil layers within a single borehole. While it is theoretically possible to back calculate the shear wave velocities for different layers with downhole seismic tests, making it the cheaper option, but in the authors' opinion, the most direct form of measurement is a better method.

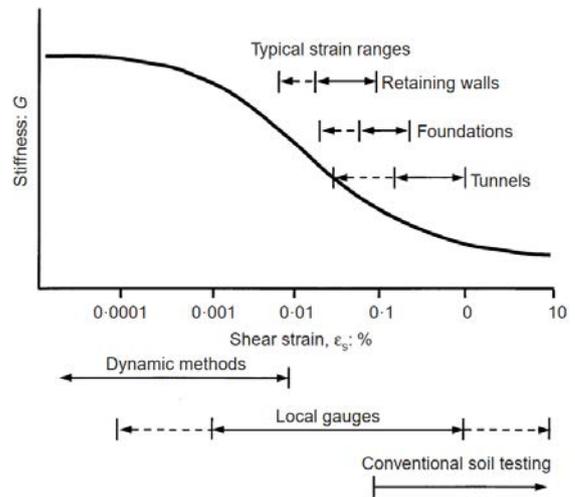


Figure 4 Characteristic stiffness–strain behaviour of soil with typical strain ranges for laboratory tests and structures (Atkinson, 2000)

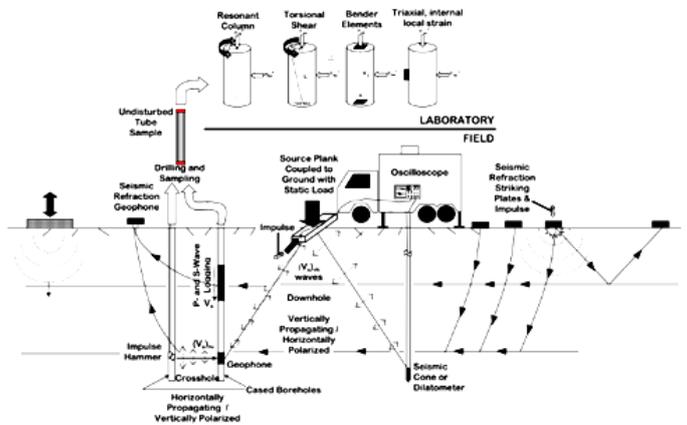


Figure 5 Field and laboratory methods to evaluate shear wave velocity (Sabatini et al., 2002)

The input parameters for the small-strain stiffness model in a typical model are as follows:

- a) G_0 – maximum small strain shear modulus
- b) $\gamma_{0.7}$ – denotes the shear strain, at which the shear modulus G is decayed to 70 percent of its initial value G_0

		Permeability k (m/s)										
		1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
Drainage	Good						Poor	Practically impervious				
Type of soil	Clean gravels	Clean sands and sand-gravel mixtures			Very fine sands, silts and clay-silt laminate			Unfissured clays and well mixed clay silts containing more than 20 % clay				
		Desiccated and fissured clays										
Recommended method of determining k	Pumping tests in situ						Flow from piezometer tips					
	Constant head permeameter tests						Equilibrium			Non-equilibrium		
	Estimation from grading curves											
		Falling head permeameter				Computed from oedometer or triaxial consolidation tests						
	Very reliable				Reliable							

Figure 2: Permeability and drainage characteristics of soil (BS8004:1986)

The above two

parameters would be able to define the entire stiffness degradation curve. The values of G_0 can be obtained from measurement of shear wave velocity from the following relationship:

$$G_0 = \rho v_s^2 \quad (1)$$

where, ρ is mass density of soil and v_s is shear wave velocity of soil.

In addition to using shear wave measurement, the maximum small strain-stiffness can also be estimated using empirical correlations. Table 1 presents the typical range for G_0 for several generic soil types. The maximum small-strain shear modulus can be correlated to the SPT N_{60} value and to the CPT q_c value as follows (Sabatini et al., 2002):

$$G_0 = 15,560 (N_{60})^{0.68} \quad (2)$$

$$G_0 = 1,634(q_c)^{0.25}(\sigma'_{vo})^{0.375} \quad (3)$$

where, σ'_{vo} is effective vertical stress of soil. Corresponding units are all in kPa.

It should be noted that empirical correlations G_0 should be treated with caution. While it may serve as an initial estimate, it is highly recommended to evaluate G_0 from shear wave velocity obtained from relevant field or laboratory tests which represents the actual site condition.

The shear strain at which the shear modulus G is decayed to $0.7G_0$ for normally consolidated soils can be calculated from the following equation (Benz, 2007):

$$\gamma_{0.7} = \frac{0.385}{4} [2 \gamma'_{0.7} (1 + 2\phi') + \gamma'_{0.7} (1 + \phi') - 2\phi'] \quad (4)$$

The values obtained above should also be checked against values given by Stokoe et al. (2004), where the authors proposed a linear increase of $\gamma_{0.7}$ from $\gamma_{0.7} \approx 1 \times 10^{-4}$ for $PI = 0$ up to $\gamma_{0.7} \approx 6 \times 10^{-4}$ for $PI = 100$. Where PI is the plasticity index of soil.

Range of typical G_0 values for various soil types are provided in Table 1. It can be observed that a large range exists between the lower and upper bound values for each soil type. Vardanega & Bolton (2011) have shown that the value of G_0 is dependent on multiple factors including PI , overconsolidation ratio (OCR), mean effective stress at point of consideration. This reiterates the point that empirical correlations solely to SPT N is subjected to limitations.

Table 1 Typical values of maximum small-strain shear modulus (Sabatini et al., 2002)

Soil Type	Maximum small-strain shear modulus, G_0 (kPa)
Soft clays	2,750 to 13,750
Firm clays	6,900 to 34,500
Silty clays	27,600 to 138,000
Dense sands and gravels	69,000 to 345,000

2.2 Important Considerations in Numerical Analyses

The use of numerical analyses such as finite element method for simulating deep excavation is common and routinely used by engineers of different levels of experience and expertise. Various constitutive soil models such as Mohr-Coulomb, Hardening Soil, Soft Soil Creep, etc. are commonly used in practice. The characteristics of each soil model varies from one another where

each may have their own advantages and disadvantages. For example, Hardening Soil model is better suited for non-linear formulation of soil behaviour in general (both soft soils and harder soil types) as compared to the Mohr-Coulomb model. However, the Hardening soil model does not capture creep (secondary compression behaviour). On the other hand, the Soft Soil Creep model is able to capture the creep or time dependent deformations; but it is only suitable for soft soils and not for other soil types.

Proper understanding of the limitations for each soil models are essential. The quality of a result from numerical analyses depends on the user's understanding of both the problem and the tool, such as the finite element software packages as well as the soil models within these software packages. One should be aware of the following factors which may affect the results of numerical analyses :

- Locations of the boundaries of the problem. The problem boundary should be located far enough away such that there is no stress rotation near the boundary. For undrained analyses, the extent of the model required will be greater.
- Details of mesh. Finer mesh is preferred to coarser meshing, especially if high strain gradients are anticipated, or for failure analyses. Nonetheless, the refinement of mesh should be carried out selectively at critical areas within the deformation mechanism to avoid unnecessarily high computation time.
- Long, thin partitions will lead to calculation instability. As such, the layout of the model and mesh should avoid these closely spaced partitions.
- Stages of construction. As soils are non-linear, stress dependent materials, thus, proper modelling of the soil at various stages from the past to its construction stages need to be carried out.
- Modelling of interfaces. Improper modelling or use of interface reduction factors may lead to dangerously unsafe design.
- Use of suitable constitutive soil models to model different geotechnical problems.
- Sensitivity of various soil parameters. For different constitutive soil models adopted in different finite element method software packages, different soil parameters may have different effects on analyses results.

3. CASE STUDIES

Three case studies including from Mass Rapid Transport (MRT) projects, with various challenging geological formations found within the Klang Valley of Malaysia; Limestone, Kenny Hill formation and Alluvium with high ground water table are discussed in the following sections.

3.1 Case Study of Deep Excavation for KVMRT in Kuala Lumpur Limestone

The Klang Valley Mass Rapid Transit (KVMRT) from Sg. Buloh to Kajang (SBK Line) is the first MRT project in Malaysia. The project comprises of a total of 9.5km long twin tunnels from Semantan to Maluri with seven (7) underground stations and associated structures such as portals, ventilation shafts, escape shafts and crossovers to be constructed over the Klang Valley and Kuala Lumpur city areas. Tun Razak Exchange (TRX) Station (known as Pasar Rakyat Station during design development), Cochrane Station and Maluri Station are underground stations located in the city area with excavation

depths of 20m to 32m deep respectively, in limestone formation. TRX Station (refer to Figure 6) is the deepest station with maximum excavation depth of 45m below ground serving as one of the underground interchange station for the KVMRT Line 2 from Sg. Buloh – Serdang – Putrajaya (SSP Line), which is currently on-going.



Figure 6 Excavation works at TRX station

The Kuala Lumpur Limestone is well known for its highly erratic karstic features. Due to the inherent karstic features of limestone bedrock, the depth of the limestone bedrock is highly irregular. Other karstic features include variable weathering condition, cavities and slime zones, which poses great challenges for construction of deep excavation works. Figure 7 shows the typical features of limestone formation (after Neoh, 1998).

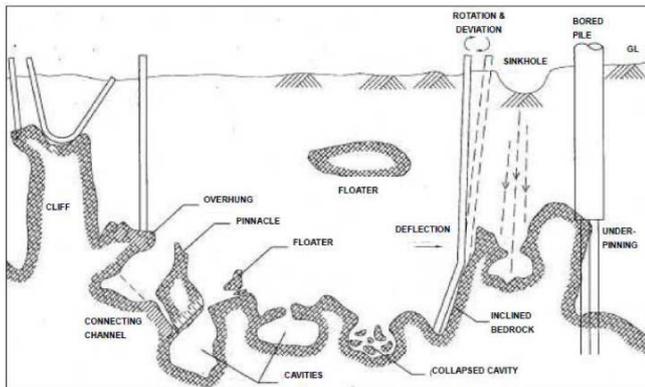


Figure 7 Typical features of limestone formation (Neoh, 1998)

Figure 8 shows the Geological Map of Kuala Lumpur (Ref: Sheet 94 Kuala Lumpur 1976 and 1993, published by the Mineral and Geoscience Department, Malaysia) superimposed with the tunnel alignment. Three underground stations namely the TRX, Cochrane and Maluri are underlain by the Kuala Lumpur Limestone.

The overburden subsoil above Kuala Lumpur limestone generally comprises of loose silty sand to sand materials with SPT^N values of less than 4. Average unit weight and permeability of subsoil are 18 kN/m³ and 1x10⁻⁵ m/s respectively. Interpreted effective shear strength from consolidated undrained triaxial tests (CIU) is $c' = 1\text{ kPa}$ and $\phi' = 29^\circ$. Bedrock profiles of limestone formation are highly variable which ranges from 3m to 30m below ground. Cavities, pinnacles and valleys are detected during subsurface investigation works.

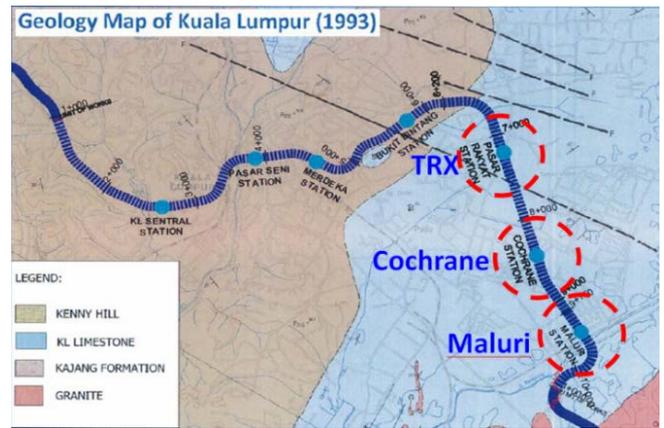


Figure 8 Geological map of Kuala Lumpur superimposed with KVMRT (SBK Line) tunnel alignment

The selection of retaining wall system for the deep excavation has considered the workability and suitability of the subsoil and rock conditions. Secant pile wall was selected as the earth retaining wall supported by temporary ground anchors. The advantages of the selected wall type are

- Water-tightness to prevent groundwater draw-down at the retained side;
- The ability to vary the pile lengths to suit the irregular limestone bedrock profiles;
- Primary pile serves as reference for reinforcement determination based on more accurate bedrock profiles. The hard/firm secant pile wall consists of primary (female) piles cast first with concrete strength class C16/20 without reinforcement and followed by secondary (male) pile with concrete strength class C32/40 with reinforcement. Figure 9 shows typical arrangement of the secant pile wall.

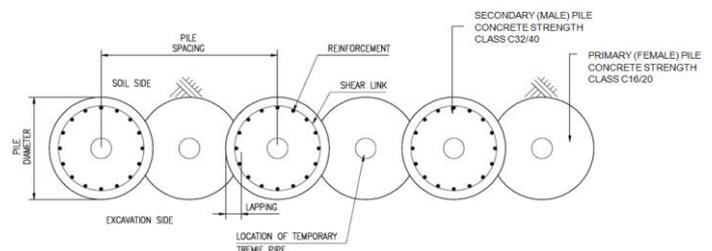


Figure 9 Typical arrangement of secant pile wall

The secant piles sizes used for this project are 880mm, 1000mm, 1180mm, and 1500mm. The secant pile was generally designed with an overlap of 15-20% of pile diameter. The extent of overlapping of the secant piles are governed by pile installation verticality, pile deviation and pile depth (Gaba et al., 2003). After reviewing the piles as-built performance, the recommended overlapping values of secant pile wall are shown in Table 2 where overlapping of up to 34% were specified to ensure watertightness of the wall.

The analyses of the retaining wall were carried out through finite element method. Wall displacement, bending moment and shear force were obtained from the analyses for structural design. A load factor of 1.4 for bending moment and shear force were applied for pile reinforcement design. The quantity of reinforcement ranges from 0.5% to 4% of pile cross-sectional area depending on the analysis based on different rock head level. 20kPa construction surcharge and 0.5m unplanned excavation were considered in ultimate limit state design. Serviceability limit state analyses were carried out to ensure the ground deformations caused by excavation will not exceed acceptable threshold limits of existing buildings and structures.

Table 2 Overlapping length of secant pile wall

Pile Diameter (mm)	Pile Length <8m (mm)	Pile Length <15m (mm)	Pile Length <25m (mm)
880	130	170	-
1000	150	200	340
1180	170	230	360
1500	225	260	380

All secant piles were founded on competent bedrock with minimum rock socket length of 1.5m to 4.0m. The termination criteria for rock socket are based on coring in competent bedrock with verification of point load index strength, $I_{s(50)} > 4$ MPa (equivalent to average UCS of 44 MPa). Should $I_{s(50)}$ falls below 4 MPa, an equivalent rock socket length can be calculated to compensate for the lower rock strength.

Given that the rock head level in limestone formation can be erratic with very steep inclined rock surface, one of the construction issues that typical arises are on the length of rock socket. To prevent any disputes, the authors recommend the following criteria where it must all be met before it is considered as rock for claiming purposes:

- Change of tools from that of soil to rock coring/excavation tools.
- Rock head level is taken as a flat rock surface (i.e. not inclined) that is verified at multiple points by means of Kelly bar, etc.
- $I_{s(50)}$ should be > 1.0 MPa

It is important to ensure that the retaining wall is socketed into competent bedrock as the vertical rock excavation is just 1.25m away from the retaining wall alignment. Support system will be installed in stages until reaching the bedrock level. A row of tie-back rock bolts was installed above the bedrock level to enhance wall toe stability. Toe stability check was carried out with some modification which replaces passive resistance by tie-back force to achieve minimum safety factor of 1.2. In addition, vertical stability was checked with resultant vertical load from ground anchor pre-stress against the rock socket length.

Excavation was carried out in stages facilitated by installing temporary ground anchors. U-turn ground anchors were used as local authority requires removal of ground anchors after construction. The anchor consists of a few pairs of strands with different unit lengths. Adopted strand diameter is 15.24mm with U-turn radius of 47.5mm. Proof tests were carried out prior to the working anchor installation for design verification. Based on the proof test results, the recommended reduction factor due to bending of strand at U-turn point is 0.65. Working loads of anchor ranged from 212kN to 1060kN with 2 to 10 strands. Typical designed pre-stress load is 60-80% of the working load capacity. Generally, the anchor will be locked off at 110% of designed pre-stress load. All anchors are subjected to acceptance test up to 125% of working load before lock-off. It is important to clearly define in construction drawing the anchor working load, pre-stress load and lock-off load to prevent misunderstanding and confusion during construction works.

The design criteria considered in strutting design are earth pressures and groundwater, material dead load, 1.5 kN/m live load, eccentric load, temperature effect (changes of 10°C), accidental impact load (50kN in vertical direction; 10kN in horizontal direction), and one-strut failure. Recommended partial load factors for strutting design are shown in Table 3.

The requirement to consider accidental and one strut failure load case is often a subject to debate. Nonetheless, the general idea is to provide for a robust design which can prevent catastrophic or progressive failure in the event of a severe but transient load case

scenario (e.g. an accident during lifting operations, etc.). It can be appreciated that this depends greatly on the anticipated so-called accidental load conditions. The above-mentioned accidental impact load was recommended by CIRIA C517 (1999) but site-specific risks and hazards may dictate different loads.

Table 3 Partial load factors

Load Case	EL	DL	LL	TL	IL
Working Condition	1.5	1.4	1.6	1.2	NA
Accidental Impact	1.05	1.05	0.5	NA	1.05
One-strut Failure	1.05	1.05	0.5	NA	NA

Note:

EL – Earth pressure and groundwater

DL – Dead load

LL – Live load

TL – Temperature effect

IL – Accidental impact load

NA – Not applicable

If the strutting design was found to be inadequate for the considered impact load, then the strut is expected to fail resulting in a one strut failure condition. The spirit of the one strut failure as stated in CIRIA C760 (2017) refers to the potential loss of support from a single prop rather than the loss of an entire level of props, unless it is considered as a physical possibility. The latter is typically simulated in a plane-strain 2D analyses where a single prop level is deactivated instead of reducing the stiffness of the props; accounting for the larger span between struts.

It needs to be recognised that a one-strut failure is in fact a 3D problem. For an intermediate strut level, a single strut failure would cause the stresses to be distributed to the closest supports; adjacent struts to its sides and to struts above and below it. The redistribution of load is governed by the vertical and horizontal stiffness of the wall, and to a lesser extent, the walers. This is because the pressure acting on the wall must first be taken by the walls before it is transferred to the waler. A weaker wall could fail even if the walers do not.

Stiffness of the wall in each direction depends on the reinforcement design and the spacing of supports. Since most walls are constructed vertically with the main reinforcement in the vertical direction coupled with generally smaller vertical spacings between levels in comparison to the horizontal spacings, vertical stiffness of the wall would typically be dominant. Hence most of the load would be transferred vertically; to the struts above and below.

If a 3D analysis is not carried out, a conservative approach to cater for this is to ensure full distribution of loads vertically. This is done by removing the strut completely in a 2D analyses. As long as the wall stresses are still within limits for the reinforcements (i.e. FOS 1.05), the resulting strut forces would then be conservative. In contrast, the force distribution would be less representative if the stiffness of the struts (intermediate strut layer) is reduced to account for a larger span for one strut failure.

The only exception for this is with regards to the top-level strut where it's failure during the initial stages (i.e. when only the single strut is installed) results in a cantilever condition. Designing for the wall to survive a cantilever condition with relatively lower risks at these shallow excavation depths would be less feasible.

To comply for a single strut failure condition, propping stiffness is reduced and waler effective length is increased accordingly. This is in-line with the concept proposed by CIRIA C760 (2017). Ultimately it would result heavier design for the walers at this top level as it must be sufficiently stiff to redistribute the loadings horizontally as it remains as the only propped level during this initial stage.

The loss of a strut single strut would effectively increase the loadings by 50% for the immediate adjacent struts. This would be mostly covered by the partial factors of about 1.5 for the working condition. Even if the strut was designed to have 100% efficiency in the basic load case, it would only need marginal increase its design to comply with the lower 1.05 partial factor for one-strut failure.

However, the same cannot be said for walers. This is because the resulting bending moment of a waler is related to the square of its effective length. This essentially means that the effective span of the waler doubles during a single strut failure, causing the bending moment to increase by 400% as opposed to the 50% increase in load for struts. The resulting waler design to comply with single strut failure would thus be significantly more conservative.

In general situations, it can be inferred that a one-strut failure load case would usually be more critical than the accidental load case. Thus, if it is considered, accidental impact condition is no longer relevant. The authors opine that if the adopted accidental impact loadings are representative of a credible worst-case scenario, then the risk of a single strut failure is as low as reasonably practical (ALARP), where designing for one-strut failure is not required.

The above discussion excludes the scenario where specific risks dictates very high accidental impact loads where it becomes less feasible to design the strutting to withstand it. In these cases, a one-strut failure check and design would be more appropriate. Nonetheless for the KVMRT project, it was a client's requirement to check for both load conditions.

It is essential to note that groundwater control is essential and it is one of the important criteria to be considered in excavation works. Groundwater drawdown may lead to excessive ground settlement and occurrences of sinkholes surrounding the excavation. Potential risk of excessive groundwater ingress into excavation pit shall be evaluated especially in limestone formation. Natural features of solution channel with cavities and highly fractured limestone connected to excavation pit may cause disastrous flooding inside the excavation pit. Therefore, grouting in limestone was carried out as a risk mitigation measure for groundwater control. Schematic of the excavation works is shown in Figure 10. Note that rock slope strengthening indicated in Figure 10 is provisional only. The actual locations and extent of rock slope strengthening are determined after geological mapping works and kinematic analyses.

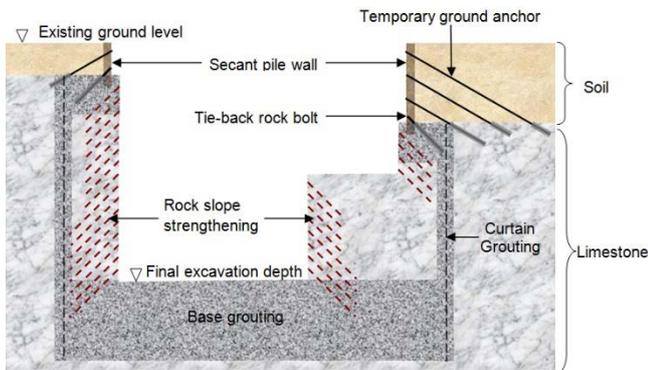


Figure 10 Schematic of excavation works

Grouting works are mainly carried out in limestone to reduce the rate of groundwater inflow into excavation and reduce pathways of water flow into excavation area. Rock fissure grouting was carried out along the perimeter of excavation area to form curtain grouting up to 10m below final excavation level. Fissure grouting involves a single packer in ascending or descending stages in order to inject grout suspension into existing pathways, fissures, cavities and discontinuities within the rock formation. Additional

grouting may be required after reviewing the grout intake from primary grouting. Rock fissure grouting is also adopted for base grouting at larger grout hole spacing. If any cavities are detected during drilling or grouting works, compaction grouting with cement mortar will be used as cavity treatment. It should be noted that grouting techniques rely much on local experiences and contractor workmanship. The recommended holding pressures for fissure grouting in limestone in this case study are shown in Table 4.

Table 4 Holding pressure for fissure grouting

Depth (m)	Holding pressure (Bar)
0 to 10	2 to 4
10 to 20	6 to 8
20 to 30	10 to 12
30 to 40	14 to 16
40 to 50	18 to 20
> 50	>22

Note: Termination criteria shall be satisfied with flow rate less than 2 litres per minute or grout volume reaches 10m³ for every grouting zone in 5m depth.

The Maluri station is located underneath one of the major public roads in town. Besides the challenging limestone bedrock, major utilities diversion (e.g. high voltage electrical cables) and traffic diversion were carried out in stages and carefully planned for installation of the secant pile wall. Temporary road decking with deck posts were installed concurrently with secant pile wall installation. Approximately 300m long and 21m wide road decking covered up the top of the station and crossover area where excavation works beneath are executed concurrently.

One of the construction difficulties was pile installation under existing electrical transmission lines with safe allowable working head room of only 13m. A modified low headroom machine was used for secant pile installation. In this condition, limit of drilling size to small diameter is required to fulfil the capacity of the modified machine. Deck posts installation required high capacity rig with deep rock drilling which is beyond the typical machine capacity. Deck posts are supported by four micropiles in a group to support the underneath existing electrical transmission line. Figure 11 shows the base slab casting at Maluri station with live traffic on top of the temporary road decking while Figure 12 shows the excavation works with strutting support at the Maluri Portal. Vibrations from close proximity rock blasting (< 3m clear distance) from the deck posts were successfully controlled below the conservative 25mm/s limit adopted for reinforced concrete frame structures.

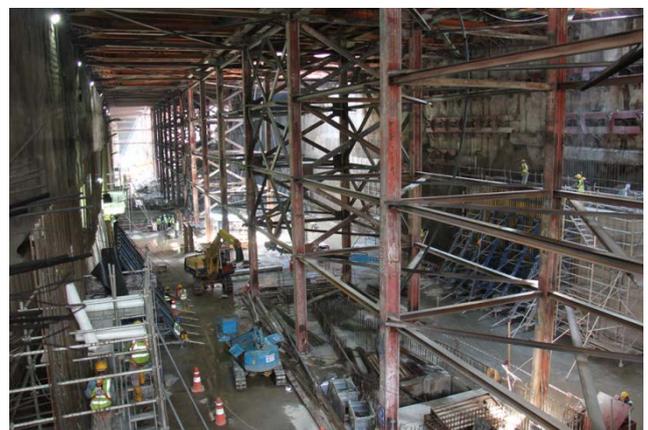


Figure 11 Maluri Station base slab casting with live traffic on top of the temporary road decking



Figure 12 Maluri Portal with strutting support

The above case study has shown the challenges of deep excavation in limestone formation in general, focusing on conventional linear or rectangular excavations. For smaller excavations such as intervention, TBM launching or escape shafts, a circular shaft will be much more economical since it transfers the lateral forces through hoop stresses. This allows for a more economical design with smaller diameter and a lighter steel reinforcement. A circular shaft would also allow for an unobstructed excavation area which translates to faster overall construction progress. It should be highlighted that in these cases, the overlapping of piles may be larger than the typical 15-20% in order to maintain effective shaft thickness after considering deviations from setting out and verticality tolerances. As a contingency, provisions of ring beams should be included for redundancy or when deviation exceeds the allowable limit. Further information on similar local experiences can be made to Tan et al. (2016) and Tan & Chow (2008).

3.2 Case Study of Deep Excavation in Alluvium with High Groundwater Table

The following is an interesting case study where a deep excavation works has caused damages to shophouses within 300m away from the edge of the excavation works. The deep excavation works were for the construction of a shopping mall with two levels of basement car park located in Penang, Malaysia. The site is situated within Quaternary deposits of marine clays, and is within the old river which has been reclaimed. Figure 13 shows the soil profile within the site, where interbedded layers silty sand can be seen in the marine clays.

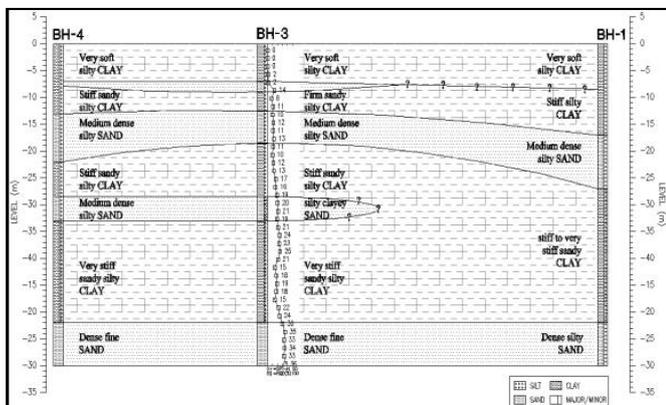


Figure 13 Soil profile within the site

The main activities near the vicinity of the affected shophouses were the basement excavation and piling works. The piles along the perimeter of the site had been driven and a temporary cofferdam using 15m deep sheet pile wall supported by internal

struts. Figure 14 shows the schematic of the temporary sheet pile wall of the original design. The plan area involved was approximately 20,000m². The general depth of excavation was 7m, while at locations of lift pits the excavation was about 10m.

By the time 40% of the area was excavated to 7m below ground level, the water level in the excavation pit has increased to about 4m below ground level. Subsequently, a stop work order was issued when settlement and damage to the shophouses were reported. Investigations were carried out to identify the causes of the issues.

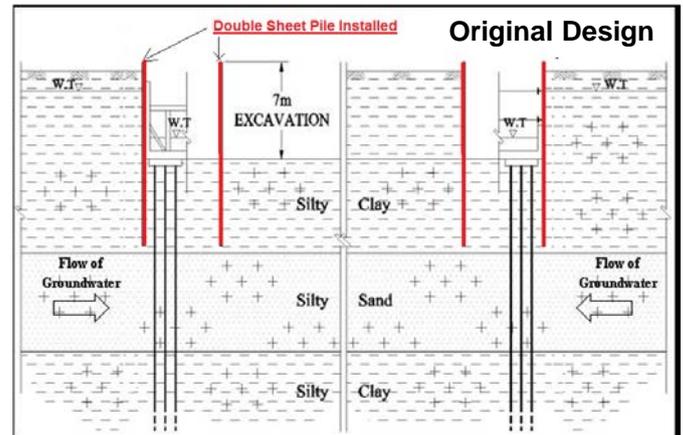


Figure 14 Schematic of the temporary sheet pile wall of the original design

During the investigation, 26 number of boreholes were drilled outside of the development area with the objectives of mapping the subsoil profile, groundwater profile and soil properties within the site. Piezometers were installed in most of the boreholes to monitor the water profile. Settlement profiles along the lines were also measured.

The results of the original ground investigation (see Figure 13) indicated the present of sand layers within the marine clay especially near the toe of the sheet piles, with a layer of sand of about 7m thick. The surface water profile during the investigation as shown in Figure 15, indicated that the groundwater on the retained side has dropped significantly, especially near the excavation. This indicates seepage through the sheet pile wall.

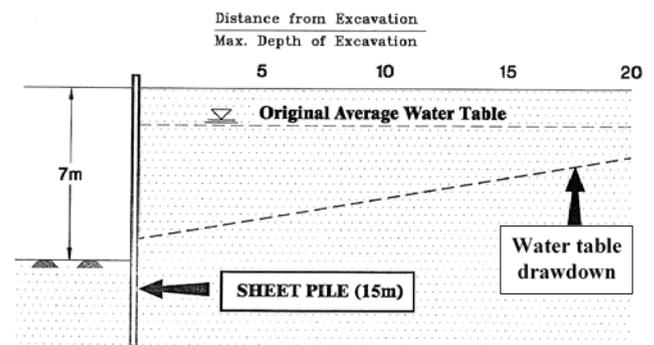


Figure 15 Groundwater profile after excavation

Every drop of a metre of groundwater would increase the effective overburden pressure by 10kPa which is equivalent to about half a metre of compacted earth fill. Thus, it causes immediate settlements and in cases of clays, consolidation settlements. In addition to that, as the drop of water level reduces with distance away from the excavation, the increase in effective overburden and hence settlements, also reduces respectively, inducing differential settlements. Ultimately, it was the differential settlement or distortion that caused the damages to the shophouses.

Remedial works were carried by installing an additional row of sheet piles of about 30m deep, penetrating into the relatively impermeable clayey soil underlying the sandy soils to serve as a hydraulic cut off in conjunction with recharging wells. These have effectively restored the water table to its original level. Further basement excavation was allowed to be resumed after the monitoring confirmed the effectiveness of the remedial measures.

4. LESSONS LEARNT

The lessons learnt based on the two case studies above are summarised below:

- (a) Sufficient and detailed SI is required to identify representative soil parameters, particularly in limestone formation where rock level is very erratic in nature.
- (b) Rock probing is crucial for the identification of any inclined rock levels, potential rock fissures and cavities which can be dealt with via grouting at an early stage.
- (c) Rock head level and socketing criteria should be stated clearly in construction drawings with prevent disputes.
- (d) Design of strutting should at least account for accidental load impacts if not one-strut failure, where deemed necessary.
- (e) A single strut failure would have significant cost implications particularly on the waler designs due to its increased effective span. Therefore, it is possible for a single layer strut failure approach to yield cheaper overall design without changing the waler span. This is not applicable for first layer struts where its failure would imply a cantilever wall condition.
- (f) Deep excavation can be carried out simultaneously with live traffic on top of the temporary road decking above the excavation.
- (g) Groundwater drawdown induced building settlement and damages could be as far away as 300m from the excavation, particularly with soft soils such as alluvium with interbedded sand layers. Proper design of hydraulic cutoff is needed to prevent excessive drop in groundwater table.

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